EFFECT OF LIMITING MOBILISED TENSION ON THE SETTLEMENT RESPONSE OF REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

by

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EFFECT OF LIMITING MOBILISED TENSION ON THE SETTLEMENT RESPONSE OF REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

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CERTIFICATE

This is to certify that the work contained in the thesis entitled EFFECT OF LIMITING MOBILISED TENSION ON THE SETTLEMENT RESPONSE OF GRANULAR FILL-SOFT SOIL SYSTEM by Sudesh Tiwari has been carried out under my supervision and that this work has not been submitted elsewhere for a degree

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NOTATIONS

B - half width of strip loading

G_t - shear modulus of upper shear layer

G_t^* - Nondimensional shear parameter of upper shear layer

G_b - shear modulus of lower shear layer

G_b^* - Nondimensional shear parameter of lower shear layer

H_t - Thickness of upper shear layer

H_b - Thickness of lower shear layer

i - Space subscript

k_f - Modulus of subgrade reaction for granular fill

k_s - Modulus of subgrade reaction for soft foundation soil

k - Lateral stress ratio

L - Half width of reinforced zone

q - Load intensity

q^* - Nondimensional load intensity

q_o - Uniform load intensity

q_t - Vertical stress at the top of membrane

q_b - Vertical stress at the bottom of membrane

q_o^* - Nondimensional uniform load intensity

T - Mobilised tensile force in the membrane per unit length

T^* - Nondimensionalised mobilised tensile force in the membrane per unit length

w - Vertical displacement

W - Nondimensional Vertical displacement

x - Distance from the centre of loaded region

X - Nondimensional distance

α - Modular ratio

θ - Slope of membrane

μ_t - Interfacial friction coefficient on the top face of membrane

μ_b - Interfacial friction coefficient on the bottom face of membrane

Chapter 1

Introduction

1.1 General Introduction

In the present era land has become a scare commodity The civil engineers are often required to construct civil engineering structures at sites which are not fit for such constructions from engineering point of view The land available may be of low bearing capacity or submerged The first remedial method for such problems which strikes to our minds is to change the site of construction As mentioned earlier due to scarcity of land or due to stratergic importance of the site such a solution is not always feasible

Hence the Civil engineers are often forced to look for alternative solutions rather than the change of site of construction This led to the developement of a technique known as *ground improvement* Ground improvement technique as its name suggests, involves the improvement of soil by various means and makes it fit for the intended purpose

The ground improvement involves various methods such as Compaction Vibroflotation, lime piles, sand coloumns, deep blasting and soil reinforcement Of all the methods mentioned above each one has its own limitations While some methods are not physically feasible for a particular problem the others may not show desired results for a particular soil type The soil reinforcement method has shown fairly good results for a wide range of problems

With the passage of time the technique of soil reinforcement has gone through major changes Earlier mainly steel was used as reinforcement but due to corrosion of steel and its high cost, alternatives had to be looked for The various other alternative reinforcements which were and are used include stainless steel, aluminium, fibre glass till the advent of most promising material in this category 'GEOSYNTHETICS' The family of geosynthetics includes geotextiles geogrids, geomembranes and geocomposites

In the last three decades, the use of geosynthetics has become a very common phenomenon in civil engineering constructions However, now a days geosynthetically reinforced-granular fill soft soil systems are frequently used as foundations for unpaved roads, embankments, shallow footings, oil drilling platforms, etc

The quality which makes geosynthetics such an important reinforcement material is its ability to restrict the developement of tensile strains in the soil The degree to which these geosynthetics modify the over all behavior of soil depends upon the orientation of the material with respect to the principal tensile strains in the

reinforcement zone, as well as on the readiness of the material to pick up the tensile stress

The construction with these inclusions is very simple, cost effective and time saving Thus they have become the first choice of designers now a days

1.2 Scope And Organisation Of Present Work

A critical review of the literature available pertaining to the reinforced foundation soil is presented in chapter 2 From the literature review it has been observed that there are various aspects such as horizontal shear stress transfer at the fill reinforcement interface, developement of mobilised tension in the reinforcement the compressibility of granular fill and the time dependent behavior resulting from the consolidation of the soft foundation soil, which need consideration while using geosynthetic reinforced granular fill soft soil system in the field and estimating its settlement under the applied load

Chapter 3 deals with the developement of foundation model using various factors stated above, the main stress has been on the mobilised tension in such reinforced systems The model also incorporates parameters like shear modulus of granular fill and the characteristics of soil geosynthetic interface. The approach used is very mechanical and simple to understand A detailed derivation of the response function of the proposed foundation model is also presented in this chapter The finite difference technique is used for solving the differential equations governing the

settlement response of the proposed foundation model

In chapter 4, the results obtained from the suggested model are presented A detailed parametric study is carried out to bring out the effect of each parameter on the settlement response of the system The results are presented in non dimensional form

Chapter 5 deals with the summary and conclusions of the present work and scope for future work

Chapter 2

LITERATURE REVIEW

2.1 Introduction

In the present era suitable land has become a scarce commodity. As mentioned earlier, due to this constraint the civil engineering constructions are to be done at sites not suitable for such constructions. The constructions on such sites are often done by improving the condition of the site by various means of ground improvement. One such technique of ground improvement is reinforcement of the foundation soil. Thus, the behaviour of reinforced soil systems is of great importance in the structural design of foundations. This interest has generated several theoretical and experimental studies in the area of reinforced soil-foundation interaction. Several concepts have been developed to explain the reinforcing mechanism of the reinforcement used in the soil. A large number of model tests have also been conducted to bring out the effects of various parameters on the load carrying capacity and the settlement characteristics of the reinforced systems using geosynthetics. This chapter presents

a detailed review of the various works, experimental as well as analytical done in the field of reinforced earth

2.2 Reinforcing Mechanism

Several concepts have been developed to define the basic mechanism of the reinforced soil The effect of inclusion of relatively inextensible reinforcement (such as, metals fibre-reinforced plastics, etc having high modulus of deformation) can be explained using either an induced stress concept or an induced deformation concept

The induced stress concept is related to an apparent cohesion (Schlosser and Vidal 1969) The tensile strength of the reinforcement and the friction at the soil-reinforcement interfaces give an apparent cohesion to the reinforced soil system At the same time, the friction mobilised at the soil- reinforcement interfaces causes a rotation of principle stresses in the soil and modifies the initial state of stresses The induced deformation concept was presented by Basset and Last (1978) This concept considers that mechanism of tensile reinforcement involving anisotropic restraint of the soil deformations in the direction of the reinforcements This results in a rotation of the principal directions of the deformation tensors Basset and Last (1978) suggested that more can be learnt by analysis of the modifications to strain fields caused by reinforcement than by study of stresses and forces Analysis of strain fields also suggested the ideal reinforcing pattern below a shallow footing The ideal pattern has reinforcement placed horizontally below the footing, which

becomes progressively more vertical farther away from the footing

If relatively extensible reinforcements, such as geosynthetics are used in the same manner as inextensible reinforcements they will also inhibit the development of internal tensile strains in the soil and develop tensile stresses However the difference between the extensible and inextensible reinforcement exists and are significant in terms of the load-settlement behaviour of the reinforced system (McGown *et al*, 1978)

At present the role of geosynthetic in load carrying capacity and settlement characteristics of the geosynthetic-reinforced foundation soils is regarded in five different ways The first is the increase of bearing capacity by changing the failure mode, i e the geosynthetics tend to force a general, rather than a local shear failure Second is the reduction of the maximum applied stress due to a redistribution of the applied surface load below the geosynthetics by providing restraint of the granular fill if embedded in it or by providing the restraint of the soft soil and the granular fill, if placed at their interface (*confinement effect*) The third is the supplementary support due to *membrane effect*, i e the deformed geosynthetic provides an equivalent vertical support (Giroud and Noiray, 1981, Love *et al*, 1987, Madhav and Poorooshasb, 1988) Geosynthetics improves the performance by acting as a separator between the soft soil and the granular fill referred to as *separation effect* (Nishida and Nishigata 1994) Use of geogrids has another benefit owing to the interlocking of the soil through the apertures of the grid membrane known as

Anchoring effect (Guido *et al.*, 1986)

2.3 Analytical Works

Binquet and Lee (1975) analysed the problem of reinforced earth slabs and presented an analytical method of designing the reinforced earth slabs for strip loading. They carried out model tests and observed that three modes of bearing capacity failure take place: (i) shear force above the upper layer of reinforcement, (ii) ties pull out and (iii) ties failure. These modes of failure were found to be dependent on the arrangement and strength of reinforcements. They also carried out cost analysis of the design and observed that if the corrosion of reinforcing material is neglected a considerable reduction in costs of the foundation can be achieved. However, if the corrosion of reinforcements are considered the savings in the cost of foundations are reduced considerably.

Giroud and Noiray (1981) presented a method for designing of unpaved roads with geotextiles at the interface between the aggregates and the subgrade soil. The procedure involved in developing this method involves three steps: (i) An empirical formula deduced from full scale tests results is related unpaved roads without geotextiles, it gives the thickness of aggregate layer as a function of traffic and soil properties. (ii) A theoretical analysis compares unpaved roads with and without geotextile, it determines the reduction of aggregate thickness resulting from the use of geotextile. (iii) by combining the empirical formula and the theoretical

analysis, the design charts were established These charts allow the determination of aggregate thickness for an unpaved road with geotextile, when traffic is taken into account It was mentioned that these charts are to be used only for saturated soils

Andrzej Sawicki (1983) developed an rigid-plastic model of reinforced earth and applied the model for determining the bearing capacity of the footing on reinforced soil They neglected the slippage on the interfaces between the soil and reinforcement They also assumed that both constituents coexist at every point of the material They also neglected the thickness of the fibers and assumed that these fibers work either in pure tension or pure compression They observed that, during the failure process of such a footing, simultaneous plastic flow of both the soil and reinforcement occured They concluded that such an rigid-plastic model for reinforced earth can be successfully applied to the analysis of various engineering problems

Sellmeijer (1990) presented a model for design of geotextiles under paved roads or parking areas on soft soils The model incorporates the membrane action as well as lateral restraint of the soil geotextile aggregate system The aggregate behaviour is modelled by an elasto-plastic shear theory, the geotextile by membrane action and lateral restraint and the subgrade soil by its bearing capacity. The incorporation of lateral restraint assures the slab effect of the aggregate whic reduces the deformations considerably Hence it was pointed out that this concept of modelling show much smaller deflections than the one where membrane action alone is considered It was

mentioned that the applicability of this method ranges from narrow low volume roads to wide parking pools

Burd and Brocklehurst (1990) analysied the mechanism of reinforcement in a plane strain reinforced unpaved road under the action of a single monotonic load The analysis was carried out considering the reinforcement stiffness as the variable parameter indicated that when geometric non linear effects are excluded variations in reinforcement stiffness have a modest effect on the load-deformation response of the road but a substantial effect on the magnitude and nature of shear stresses acting at reinforcement-soil interface The results indicated a 15% improvement in the bearing capacity of thre footing for the most stiff reinforcement However it was mentioned that the variations in reinforcement stiffness have a more significant effect on the magnitude of the shear stress acting on the lower and upper surfaces of the reinforcement

Poorooshasb (1991) studied the effect of subsidence of ground on the load settlement response of the reinforced compacted fill layer He treated the above mentioned problem as an equilibrium problem in contrast to the earlier works which treated this as an instability problem The study was carried out to investigate the various factors e g the ground subsidence, the degree of compaction of the fill and the deformation properties and strength of geosynthetic upon the performance of the system The general assumptions made during the analysis are, (i) reinforced is placed between the granular fill and the subgrade, (ii) the reinforcement is rough

enough so that there is no slippage between the fill and the reinforcement at the interface, (iii) all the vertical planes in the unloaded region remains both plane and vertical after the loading has been imposed, and (iv) the granular material possesses a constant ultimate void ratio independent of the normal stress component σ It was pointed out that the analysis is not exact, utilising only a kinematically admissible displacement field

Dixit and Mandal (1993) used variational method to determine the bearing capacity of geosynthetic reinforced soil For analysis a two dimensional plane strain problem was considered The analysis established the rupture surface and the normal stress distribution that satisfies the condition of limiting equilibrium and yields the lowest possible value for the the applied load It was assumed that the failure of reinforcement was either by breaking of the reinforcement or by slipping out of the reinforcement It was also assumed that the geosynthetic reinforcement does not alter the shear parameters of the soil The analysis lead to the following results, (i) The shape of critical rupture surface is a log spiral The shape of the critical rupture surface depends on cohesion and the angle of internal friction of the granular fill, (ii) the approach is valid only for shallow reinforcement, and (iii) the bearing capacity increases significantly with the introduction of reinforcement

Ghosh and Madhav (1994) presented a three parameter mathematical model to account for the membrane effect of a reinforcement layer on the load-settlement response of the reinforced granular fill soft soil system The model incorporated the

non linear loading pressure-settlement response of the granular fill and soft soil A detailed parametric study was carried out for a uniformly loaded strip footing which indicated that the membrane action of reinforcement causes reduction in settlement below the footing which is over and above the effect of granular fill

Ghosh and Madhav (1994) presented a mathematical model for analysis of reinforced foundation bed by incorporating the confinement effect of a single layer of reinforcement Empirical expressions were used to determine the shear modulus due to the confining effect of reinforcement In this analysis, plane strain conditions and nonlinear load settlement response of soft soil are considered From the analysis it was observed that the parametric results are sensitive to the methods used for obtaining the modified shear stiffness of the granular fill It was also noted that the confinement effect is more pronounced when the shear stiffness of the granular fill is large and the increase in the modified shear stiffness below the centre of footing is two to five times the initial values

Espinoza (1994) derived an general expression for evaluating the the increase in bearing capacity of the reinforced soil systems due to membrane action based on strict equilibrium conditions The general expressions derived proves to be helpfull in comparing the existing expressions It was noted that independently of the model used and geotextile deformation shaped assumed, comparable values for membrane support for relatively small rutting ratios are obtained However for large rutting ratios, the choice of membrane support model and geotextile deformation shape have

significant influence on the results

Shukla and Chandra (1994) proposed a mechanical model for reinforced granular fill soft soil system They incorporated the compressibility of the granular fill by attaching stiff Winkler springs at the bottom of Pasternak shear layer The equations governing the settlement response of the system were developed by considering equilibrium of the various elements of shear layer and the membrane Finite difference scheme was used to solve the governing equations The results indicated that the compressibility of granular fill has an appreciable influence on the settlement response of the system as long as the stiffness of granular fill is less than approximately 50 times, that of the soil.

Raghavanendra *et al* (1996) extended the work done by Binquet and Lee for design of reinforced soil bed as two layered system Analysis was carried out on a footing of known width resting on the surface of a two layered soil system, consisting of strong upper layer overlaying a weak soil The upper layer was reinforced in horizontal layers of different depths the non dimensional parameters for estimating the mobilization of tension in reinforcement were obtained with the help of stress distribution on the plane of reinforcement obtained from finite element analysis The results indicated that the non dimensional parameters for two layered soil system are distinctly different from that of single layer, this change in non dimensional parameter will yield higher values of bearing capacity of the two layer soil system It was observed that using a stiffer layer of soil over soft clay improves the bearing

capacity of the system

2.4 Experimental Works

Binquet and Lee (1975) carried out model tests with strip footings on reinforced sand foundations to investigate the mechanism and potential benefits of using reinforced earth slabs to improve the bearing capacity of granular soil The model tests were carried out for three different foundation conditions, (i) homogenous deep sand, (ii) sand above an extensive layer of very soft material and (iii) sand above a finite size pocket of very soft material It was observed that if all the other conditions remain the same the load settlement and the ultimate bearing capacities of the footings can be improved 2–5 times above the same for unreinforced soil It was also noted that the bearing capacity continued to improve for with increasing number of layers upto at least six to eight, beyond which very little additional improvement was observed

Akinmusuru and Akinbolade (1981) conducted model tests with square footings on a deep homogenous sand bed reinforced with flat strips of the rope fibre material The tests were carried out to determine the effects of , (i)vertical spacing, (ii) horizontal spacing and (iii) number of layers of the reinforcement on the bearing capacities of the reinforced soil systems The results showed that depending upon the arrangement of reinforcement the ultimate bearing capacity values can be improved by a factor three times that of the unreinforced soil It was however mentioned that the use of rope fibre material as reinforcement has its own limitations when the

water table rises upto the level of reinforcement There is also danger of these fibers being attached by termites

Fragaszy and Lawton (1984) conducted a series of laboratory model tests to determine the influence of soil-density and the length of reinforcing strip on the load-settlement behaviour of reinforced sand The term bearing capacity ratio (BCR) given by BCR= $\frac{q}{q_o}$ was used to express and compare the test results, where q and q_o are the bearing pressures for the reinforced and unreinforced soil respectively, at the same initial dry density and at a given settlement The results indicated that when the bearing capacity ratio is calculated at a settlement equal to 10% of the footing width, the BCR is independent of soil density It was also observed that as the strip length increases from three to seven times the footing width the BCR increases rapidly However, additional strip length does not appear to significantly affect the BCR

Guido *et al* (1985) conducted laboratory model tests to study the bearing capacity of shallow foundations reinforced with geotextiles The tests were carried out by varying the vertical spacing of reinforcement layers, the number of layers, the width size of the sheet of geotextile and the tensile strength of geotextile The results showed that the bearing capacity of the soil reinforced with geotextiles increased by a factor greater than three

Verma and Char (1986) conducted laboratory tests to determine the efficiency

of vertical reinforcements in improving the sand subgrades The tests were carried out to study the effect of extent of reinforcement spacing of reinforcement and the flexibility of the reinforcing elements on the bearing capacity and load-settlement behaviour of reinforced sand The results showed that for a given spacing of reinforcements (density) the bearing capacity is a function of the diameter and roughness of reinforcements while for a given type of reinforcement used, bearing capacity increases with the increase in density of reinforcement

Love *et al* (1987) carried out physical model tests on reinforced granular fill soft soil system Monotonic loading was applied through rigid footing under plane strain conditions to study the failure mechanisms of the system The results from the model tests were used to calibrate the analytical method The analytical modelling was done using a finite element computer program capable of allowing inclusion of a thin reinforcing layer and to handle the large deflections From the analytical and model studies it was observed that the geogrid reinforcements tends to reduce the shear stresses transmitted to the clay surface It was also observed that the failure mechanisms in clay are mobilised at quite small deformations of the fill The reinforcement has to be stiff enough and strong enough to take the tension induced by the shear stresses from the granular layer above without failing

Mandal and Shah (1992) conducted laboratory model tests to study the effect of geogrid reinforcements on the bearing capacity of clay subgrades It was observed that there was an significant increase in the bearing capacity due to the reinforcement

action of the geogrid The soil has very low tensile resistance and its tensile resistance improves with the effective bond due to interlock at the soil reinforcement interface The results showed that the maximum percentage reduction in the settlement with the use of geogrid reinforcement below the compacted and saturated clay is about 45% and it occurs at a distance of 0 25B from the base of the square foundation

Khing *et al* (1993) conducted laboratory model tests to determine the bearing capacity of a strip foundation supported by a sand layer reinforced by geogrid layers Based on the model tests the following conclusions were drawn (i) The maximum benefit of geogrid reinforcement in increasing the bearing capacity was obtained when the ratio of the depth of first reinforcing layer to the foundation width was less than unity, (ii) for maximum benefit, the minimum width of the geogrid layers should be about six times the foundation width, and (iii) the bearing capacity ratio calculated on the basis of limited settlement appears to be about 67-70% of the ultimate bearing capacity

Omar *et al* (1993) conducted laboratory model tests to determine the ultimate bearing capacity of strip and square foundations supported by sand reinforced with geogrid layers The model test results were used to determine the critical depth of reinforcement and the dimensions of the geogrid layers for mobilising the maximum bearing capacity ratio. The results indicated that for the development of maximum bearing capacity, the effective depth of reinforcement is about 2B for strip foundations and 1 4B for square foundations It was also observed that for

mobilization of maximum bearing capacity ratio the width of reinforcement layers should be about 8B for strip foundations and 4 5B for square foundations These test results cannot be applied directly to full size foundations as scale effects have not been considered during the study

Yetimoglu *et al* (1994) carried out model tests as well as finite element analysis to determine the bearing capacity of rectangular footings on geogrid reinforced sand The tests were carried out to investigate the effect of the depth of first layer of reinforcement, vertical spacings of the reinforcement layers, number of reinforcement layers and the size of reinforcement sheets on the bearing capacity The experimental and analytical studies indicated that there was an optimum embedment depth at which the bearing capacity was the highest when single layer reinforcement was used This optimum embedment depth was given as 0 3 of the footing width It was also observed that there was an optimum spacing for multi layered reinforcement at which the bearing capacity was maximum and was found to be im between 0 2B and 0 4B

Nishida and Nishighata (1994) investigated the effect of the properties of geotextile on the seperation function and the relationship between the reinforcement and seperation function in road construction The tests were carried out in two series under the action of cyclic loads In the first series, the effects of the properties of geotextiles on the quantity of fine particles movement through the geotextiles were considered In the second series of tests, interaction between the reinforcement and

the seperation function is considered It was observed from the first series of tests that the unit weight of geotextiles is applicable to estimate the seperation function The results from the second series of tests indicated that the reinforcement is a prime function when the ratio of the applied stress te the shear strength of the subgrade soil (σ/c_u) has a high value However, the separation is a more important factor when the ratio has a low value

Floss and Gold (1994) carried out field tests and finite element analysis to determine the improvement of the bearing capacity and deformation behaviour due to a geosynthetic reinforcement placed at the base of a layer of granular fill on the surface of a soft clay The study was carried out by varying the manner of application og geosynthetics (wovens, non-wovens and geogrids) and thickness of the granular fill It was observed that by using material parameters exclusively from laboratory tests a very good accordance of the FEM calculations with the sites could be established The reason for the improvement in the bearing and deformation capacity of the reinforced system was attributed to the better load spreading capacity of the geosynthetics

B M Das and E C Shin (1994) conducted laboratory test to determine the permanent settlement of a surface strip foundation supported by geogrid-reinforced saturated clay subjected to low frequency cyclic load The tests were conducted by initially subjecting the foundation to an allowable static load and then cyclic load was superimposed upon it The results indicated that (i)for a given amplitude

of the cyclic load intensity, the maximum settlement increases with the increase in the magnitude of static load intensity, (ii) for a given intensity of static loading the maximum permanent settlement increases with the amplitude of cyclic load intensity

B M Das *et al* (1996) carried out laboratory model tests for determining the ultimate bearing capacity of a surface strip foundation on a saturated clay slope reinforced with a layer of geogrid Biaxial type of geogrid was used for the test The tests were carried out by varying the location of the top of the geogrid layer, center-to-center spacing of the geogrid layer, and the depth of geogrid reinforcement The results showed that (i)other conditions remaining same, the first layer of geogrid should be located at a depth of 0 4B below the foundation for maximum increase in th e bearing capacity derived from reinforcement, (ii)the maximum depth of reinforcement which contributes to bearing capacity improvement is about 1 72B, where B is the width of footing

Adams and Collin (1997) carried out 34 large scale model tests to evaluate the effects of single and multiple layers of geosynthetic reinforcement placed below shallow footing The tests were carried out on two different types of geosynthetics a stiff biaxial geogrid and a geocell The tests were carried out by varying different parameters such as, number of reinforcement layers, spacing between reinforcement layers, the depth to the first reinforcement layer plan area of the reinforcement. the type of reinforcement, and the soil density. The results indicated that the use of

geosynthetic reinforced soil foundations may increase the ultimate bearing capacity of shallow spread footing by a factor of 2 5

2.5 Conclusions

In the previous sections, a critical review of available literature pertaining to the near surface reinforced foundation soil are presented From the results of large number of model tests conducted till very recently and also from the results presented through several analytical studies of geosynthetic reinforced granular fill soft soil system

Hence, it was concluded that there was a need to develop a foundation model which incorporates the effect of mobilised tension in a simple way so that one can easily estimate the settlements response of the reinforced granular fill soft soil system by considering most of the factors governing the behaviour under specified field conditions

Chapter 3

PROBLEM DEFINITION AND FORMULATION

3.1 Introduction

In the present chapter, a mechanical foundation model for the reinforced granular fill soft soil system is developed to incorporate various limits of mobilised tension which governs the settlement response of the system The various sub-systems of the geosynthetic reinforced granular fill soft soil system are idealised by various mechanical elements generally used to solve such soil structure interaction problemns in geotechnical engineering The equations which govern the settlement response of the system are developed by considering the equilibrium of shear layer and the rough elastic membrane The numerical solution is obtained by iterative finite difference scheme The boundary conditions and the convergence criterion used to obtain the solution are stated at the end of the chapter

3.2 Definiton And Formulation Of The Problem

A geosynthetic reinforced granular fill on soft foundation soil is shown in Fig (3 1) The foundation consists of granular fill overlying soft foundation soil A layer of geosynthetic is provided as reinforcement in the granular fill

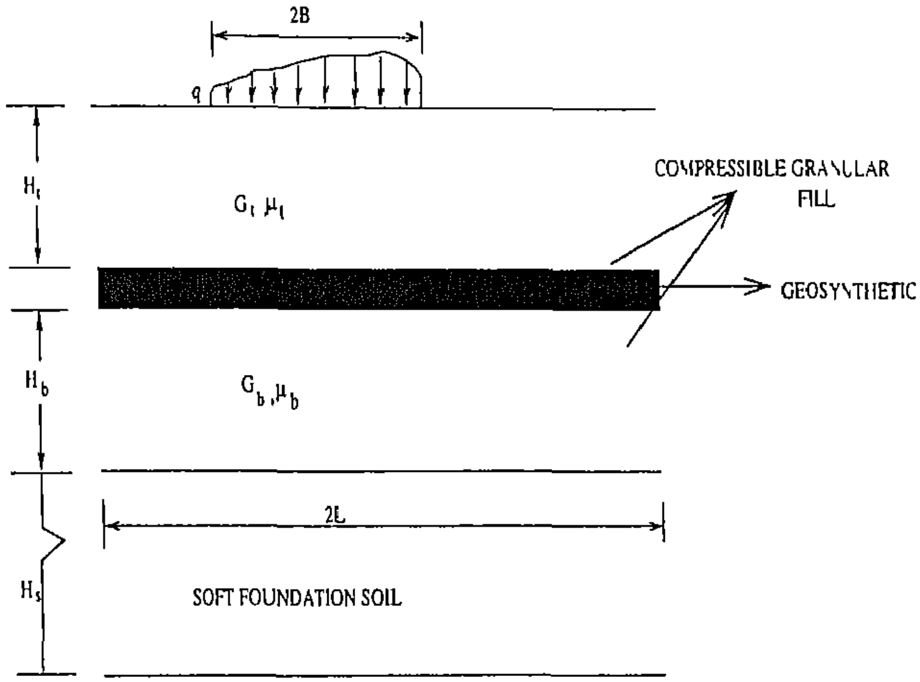

Figure 3 1 Geosynthetic Reinforced Granular Fill Soft Soil System

To idealise the above mentioned foundation system a mechanical model is proposed as shown in Fig (3 2) The granular fill is represented by Pasternak shear layer The geosynthetic reinforcement is represented by an rough elastic membrane. The soft soil is represented by Winkler springs of spring constant k_s per unit area The compressibility of the granular fill is represented by a layer of stiffer Winkler spring attached directly to the bottom of Pasternak shear layer of spring constant k_f

per unit area The spring layers connected in series to represent the compressibility of the granular fill and the soft soil are idealised by equivalent spring layer as shown in Fig (3 2)

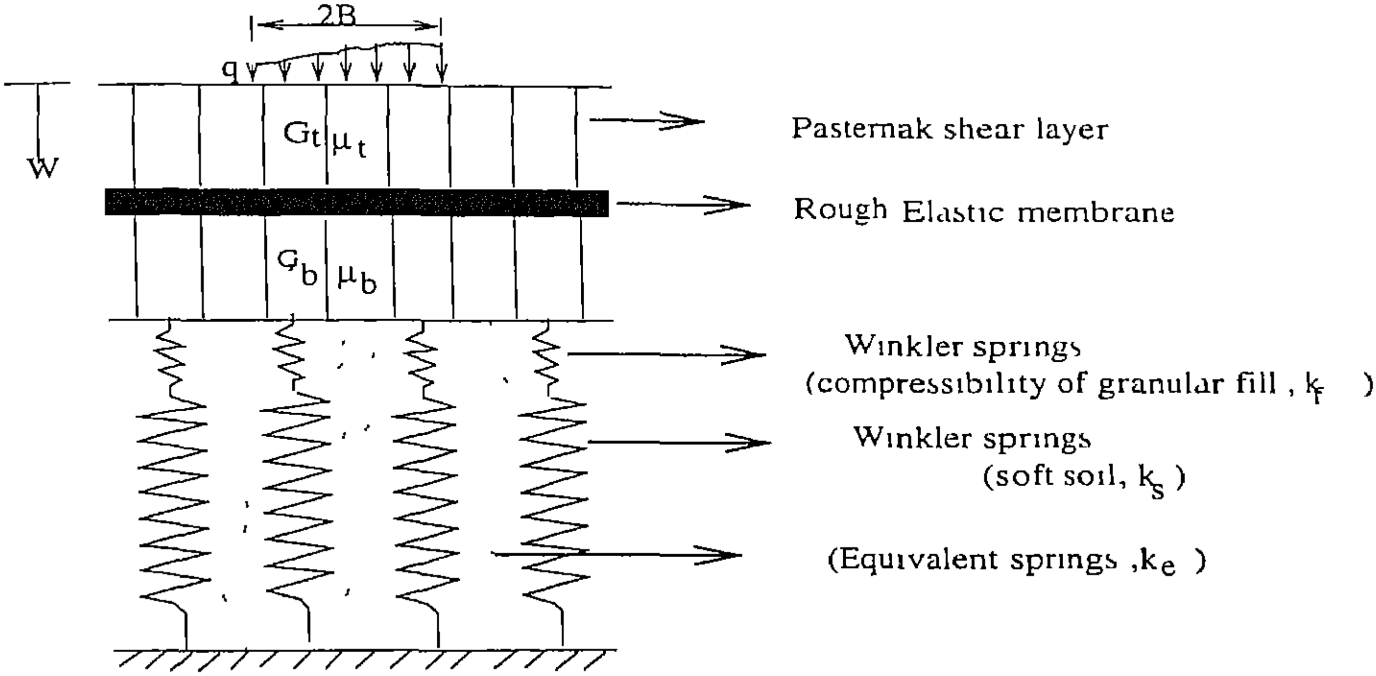

Figure 3 2 Proposed Foundation Model

To incorcoprate the various limits of mobilised tension the behaviour of the tension mobilised at the fill geosynthetic interface is shown in Fig (3 3) is used

The equation governing the response of the proposed model is derived by considering the equilibrium of forces on shear layer and the rough elastic membrane

With reference to Fig (3 4) the vertical force equilibrium equation of the upper shear layer element is given by Selvadurai as

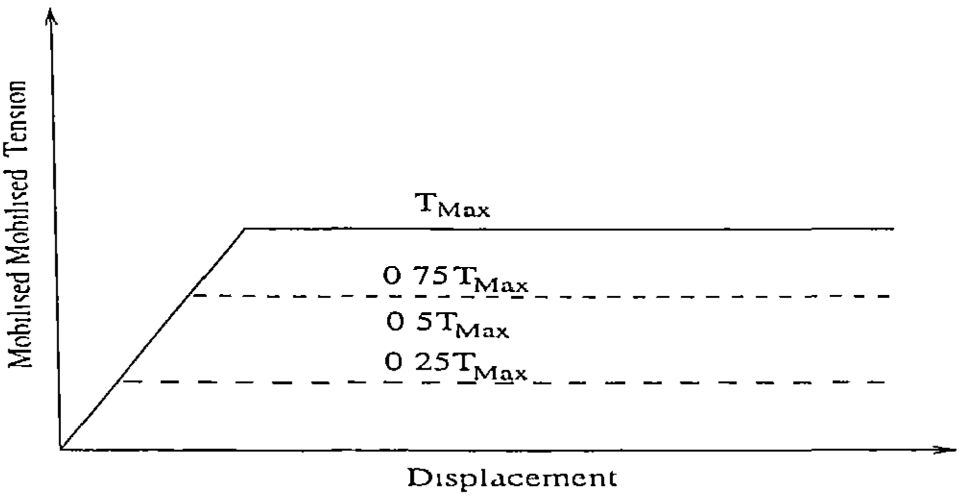

Figure 3 3 Behaviour Of Mobilised Tension

$$q = q_t - G_t H_t \frac{d^2 w}{dx^2} \tag{3 1}$$

where q is the applied load intensity, q_t is the vertical force interaction between the membrane and the upper shear layer, w is the displacement, H_t is the thickness of upper shear layer above the membrane and G_t is the shear modulus x is the distance measured from the centre of loading

The horizontal equilibrium equation of the rough elastic membrane element can be written as

$$(T + \Delta T)cos(\theta + \Delta\theta) - Tcos\theta + (\mu_t q_t + \mu_b q_b)\Delta x + k(q_t - q_b)\Delta x tan\theta = 0 \tag{3 2}$$

where q_b is the vertical force interaction between the membrane and the lower shear layer. μ_t and μ_b are respectively the interfacial frictional ratios at the top and bottom faces of the membrane, k is the coefficient of lateral stress, θ is the slope of

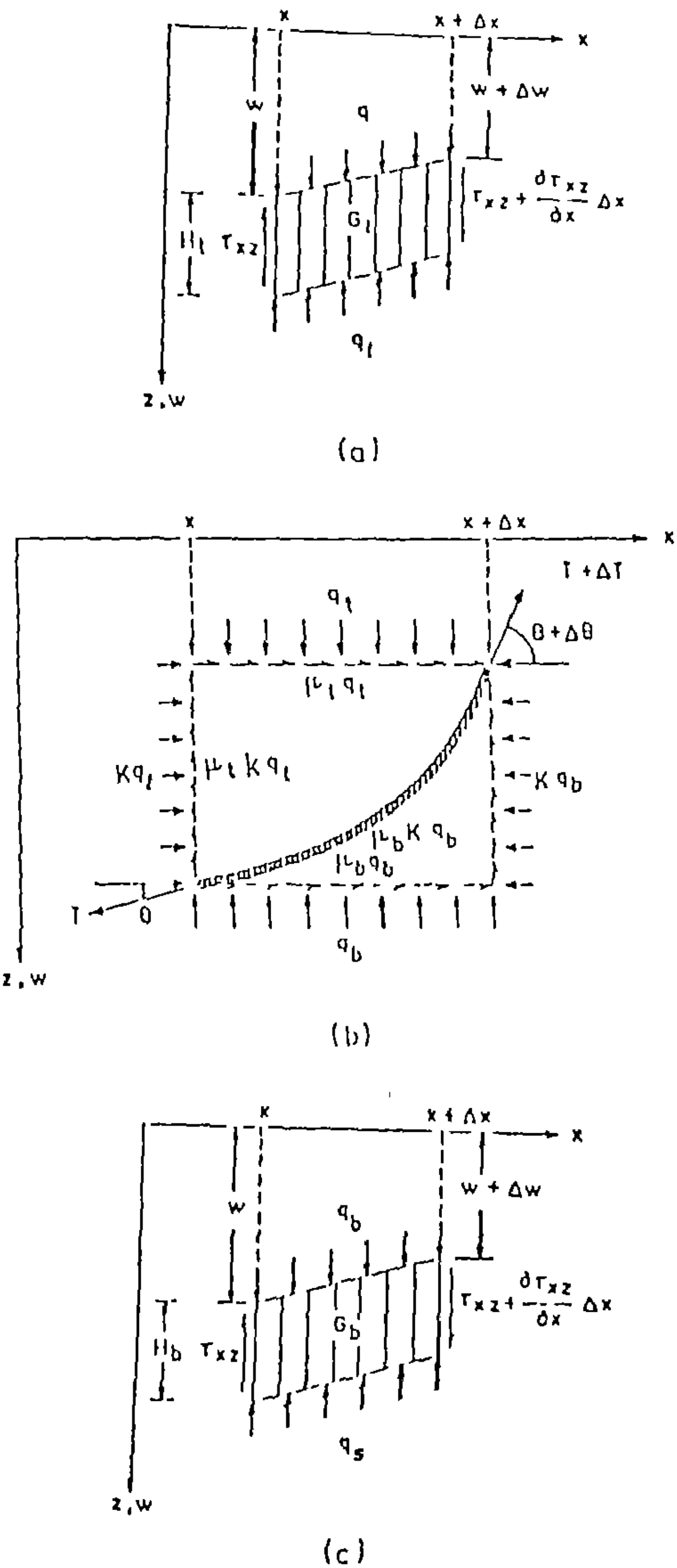

Figure 3 4. Definition Sketch (a)Forces On The Upper Shear Layer Element (b)Forces On The Membrane Element (c) Forces On The Lower Shear Element

membrane and T is the tensile force mobilised per unit length in the membrane

As $\Delta x \to 0$, Eqn (3 2) reduces to

$$cos\theta\,\frac{dT}{dx} - T sin\theta\,\frac{d\theta}{dx} = -(\mu_t q_t + \mu_b q_b) - k(q_t - q_b)tan\theta \tag{3 3}$$

Similarly the equation for vertical force equilibrium for the rough elastic membrane can be written as

$$sin\theta\,\frac{dT}{dx} + T\cos\theta\frac{d\theta}{dx} = -k(\mu_t q_t + \mu_b q_b)tan\theta + (q_t - q_b) \tag{3 4}$$

from Eqns (3 3) and (3 4)

$$q_t = q_b + \frac{Tsec\theta}{1 + ktan^2\theta}\,\frac{d\theta}{dx} - \frac{(1-k)(\mu_t q_t + \mu_b q_b)}{1 + ktan^2\theta}tan\theta \tag{3 5}$$

Substituting for $\frac{d\theta}{dx}$ in terms of vertical displacement, w into eqn(3 5)

$$q_t = X_1 q_b - X_2 T cos\theta\frac{d^2w}{dx^2} \tag{3 6}$$

where

$$X_1 = \frac{1 + k\,tan^2\theta - (1-k)\mu_b\,tan\theta}{1 + k\,tan^2\theta + (1-k) + \mu_t tan\,\theta} \tag{3 7}$$

$$X_2 = \frac{1}{1 + k\,tan^2\theta + (1-k)\mu_t\,tan\theta} \tag{3 8}$$

The vertical force equilibrium equation of lower shear elemement can be written as

$$q_b = q_s - GbH_b \frac{d^2 w}{dx^2} \tag{3 9}$$

where q_s is the vertical force interaction between the lower shear layer and the saturated foundation soil, G_b and H_b are respectively the shear modulus and thickness of lower shear layer

The expression for q_s is given by

$$q_s = k_e w \tag{3 10}$$

where k_e is the spring constant per unit area of the equivalent spring attached and is given by

$$k_e = \frac{\alpha}{1+\alpha} k_s \tag{3 11}$$

Where α is the modular ratio ($\alpha = k_f/k_s$)

Combining Eqns (3 1) and (3 4)-(3 10)

$$q = X_1 \frac{\alpha}{1+\alpha} k_s w - (G_t H_t + X_2 T cos\theta + X_1 G_b H_b) \frac{d^2 u}{dx^2} \tag{3 12}$$

The equation for the variation of mobilised tension in the membrane is determined by combining Eqns (3 1), (3 3), (3 4), (3 6)-(3 9) and is given by

$$\frac{dT}{dx} = -X_3(q + G_t H_t \frac{d^2 w}{dx^2}) - X_4(\frac{\alpha}{1+\alpha} k_s w - G_b H_b \frac{d^2 u}{dx^2}) \tag{3.13}$$

where

$$X_3 = \mu_t \cos\theta(1 + k\tan^2\theta) - (1-k)\sin\theta \tag{3.14}$$

$$X_4 = \mu_t \cos\theta(1 + k\tan^2\theta) + (1-k)\sin\theta \tag{3.15}$$

For analysis, first the value of the maximun tension mobilised at the fill geosynthetic inteface is calculated using Eqn. (3.13). Then subsequent analysis are done by limiting the value of the tension mobilised to some fixed percentage of the maximum tension mobilised as calculated above and discarding Eqn. (3.13).

3.3 Method Of Solution

The governing Eqns (3.12) and (3.13) are stated in their non dimensional form as,

$$q^* = X_1 \frac{\alpha W}{1+\alpha} - (G_t^* + X_2 T^* \cos\theta + X_1 G_b^*) \frac{d^2 W}{dX^2} \tag{3.16}$$

$$\frac{dT^*}{dX} + X_3(q^* + G_t^* \frac{d^2 W}{dX^2}) + X_4(\frac{\alpha W}{1+\alpha} - G_b^* \frac{d^2 W}{dX^2}) = 0 \tag{3.17}$$

where, $X = x/B$, $W = w/B$, $G_t^* = G_t H_t / k_s B^2$, $G_b^* = G_b H_b / k_s B^2$, $q^* = q/k_s B$, $T^* = T/k_s B^2$, $\alpha = k_f/k_s$ and B is the half width of loading

Writing Eqns (3 16) and (3 17) in finite difference form for an interior node i

$$q_i^* = X_{1_i}\frac{\alpha W_i}{1+\alpha} - (G_t^* + X_{2_i}Tcos\theta_i + X_{1_i}G_b^*)(\frac{W_{i-1} - 2W_i + W_{i+1}}{\Delta X^2}) \qquad (3\ 18)$$

$$\begin{aligned} T_i^* &= T_{i+1}^* + \frac{\Delta X}{4}[(X_{3_i} + X_{3_{i+1}})\{(q_i^* + q_{i+1}^*) + G_t^*(\frac{d^2W}{dX^2}|_i + \frac{d^2W}{dX^2}|_{i-1})\} + (X_{1_i} + X_{1_{i+1}}) \\ &\{\frac{\alpha(W_i + W_{i+1})}{1+\alpha} - G_b^*(\frac{d^2W}{dX^2}|_i + \frac{d^2W}{dX^2}|_{i+1})\}] \end{aligned} \qquad (3\ 19)$$

For analysis, first the value of the maximum tension mobilised at the fill geosynthetic inteface is calculated using Eqn (3 19) Then subsequent analysis are done by limiting the value of the tension mobilised to some fixed percentage of the maximum tension mobilised as calculated above and discarding Eqn (3 19) In Eqn (3 19) the derivatives d^2W/dX^2 has been expressed by central finite difference scheme while dT^*/dX has been expressed by forward difference scheme hence in order to minimise the numerical error, average values of q^*, W, d^2W/dX^2, X_3 and X_4 have been taken in Eqn (3 19)

3.3.1 Boundary Conditions

The solutions are obtained for a uniform load intensty acting over a width 2B, as shown below in Fig (3 5)

As there is symmetry about the centre of the loaded region the slope at the centre is taken as zero At the edge of reinforcement (i e at X=L/B, L being the

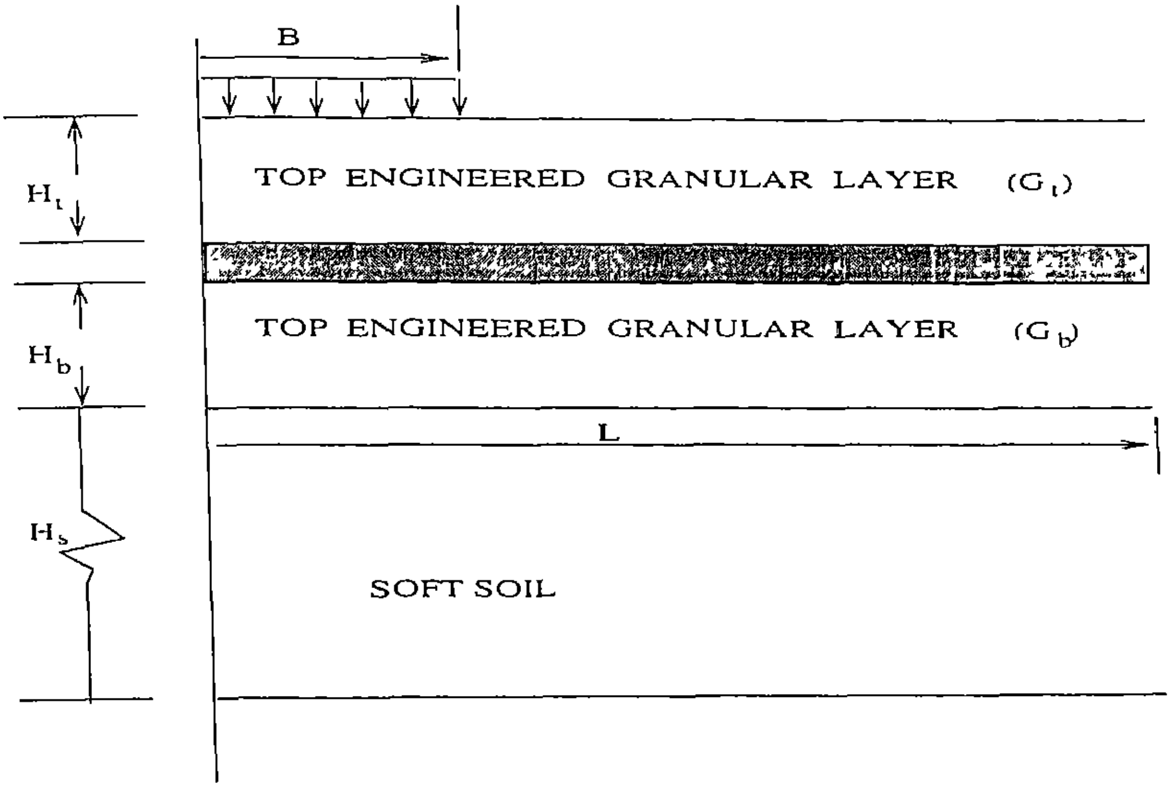

Figure 3 5 Definition Sketch Boundary Conditions And Load Analysed

half width of reinforced zone) the slope is considered as zero as observed in most practical cases whether the reinforcement is fixed or free The mobilised tension force is considered zero at the edge of reinforcement (i e $T^*=0$ at X=L/B) which implies that frictional resistance mobilised over the length of the membrane is just sufficient to balance tensile force

3.3.2 Convergence Criteria

The solutions have been obtained with a convergence criterion(Chapra and Canale 1989)

$$\left|\frac{W_i^k - W_i^{k-1}}{W_i^k}\right| \leq \epsilon_s \tag{3.20}$$

for all i, where k and k-1 are respectively the present and previous iterations ϵ_s is specified tolerance which in present case is taken as 0.000001

The ranges of various non dimensional parameters studied are shown in Table (3.1)

Sl NO	NONDIMENSIONAL PARAMETERS	RANGES
1	Load intensity, q^*	0.1-1.0
2	Shear parameter, G_t, G_b	0.01-1.0
3	Interfacial frictional coefficient, μ_t, μ_b	0.1-1.0
4	Width of reinforcement, L/B	2.0-3.0
5	Lateral stress ratio k	0.4-1.0
6	Modular ratio $\alpha(\frac{k_l}{k_i})$	5-∞

Table 3.1 Ranges Of Nondimensional Parameters Studied

3.4 Conclusions

In this chapter, the equations governing the settlement response of the system have been developed. Finite difference scheme has been used to solve the governing equations The boundary conditions and the convergence criterion used in solving the governing equations have been presented in the chapter

Chapter 4

RESULTS AND DISCUSSION

4.1 Introduction

In this section the results obtained from the model prescribed in chapter 3 are presented The results are presented in a form such as to show the effect of different parameters on the settlement response of the system The results are presented for various limits of mobilised tension developed at the fill-geosynthetic interface The effect of limiting the mobilised tension is studied by limiting the mobilised tension to some fixed percentage of the maximum mobilised tension In this study, results are obtained for the mobilised tension values limited at 75%, 50% and 25% of the maximum mobilised tension and for the case when the tension mobilised is not limited

4.2 Effect Of Mobilised Tension For Different Load Intensities

In this section the effect of limiting the mobilised tension for different values of load intensity on the settlement response of the reinforced granular fill soft soil system is shown In Fig (4 1) the settlement profiles at load intensity $q^* = 0\ 1$ $k = 1\ 0$, L/B=2 0,$\alpha = 5\ 0$ and $\mu_t = \mu_b = 1\ 0$ is shown Four different curves are presented one for the maximum mobilised tension and others for the mobilised tension limited to 75%, 50% and 25% of the maximum mobilised tension

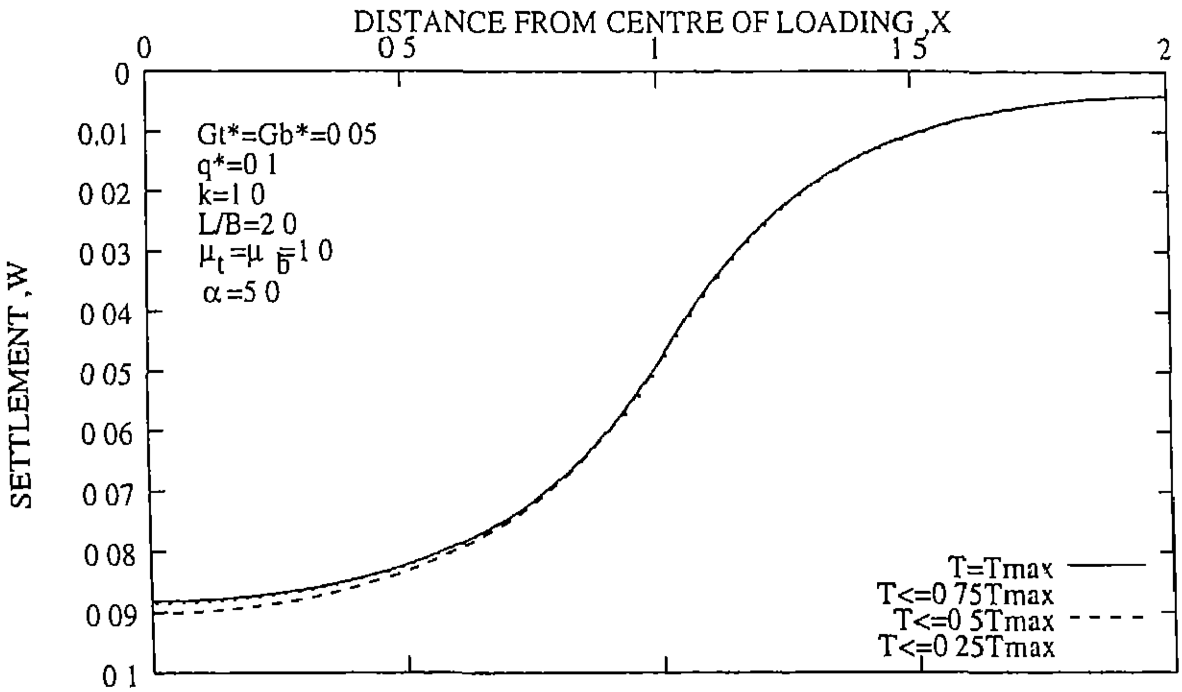

Figure 4 1 Settlement Profiles For $q^* = 0\ 1$ For Different Mobilised Tensions

From the figure it can be seen that the effect of varying mobilised tension is predominant only in the region upto L/B=0 75 Beyond this the effect of varying

the mobilised tension is negligible The increase in settlement due to varying the mobilised tension is maximum at the centre of loading The percentage increase in the settlement at the centre of loading is 0 5%, 2 2% and 5 08% respectively for the cases when the mobilised tensions are limited to 75%, 50% and 25% of maximum mobilised tension

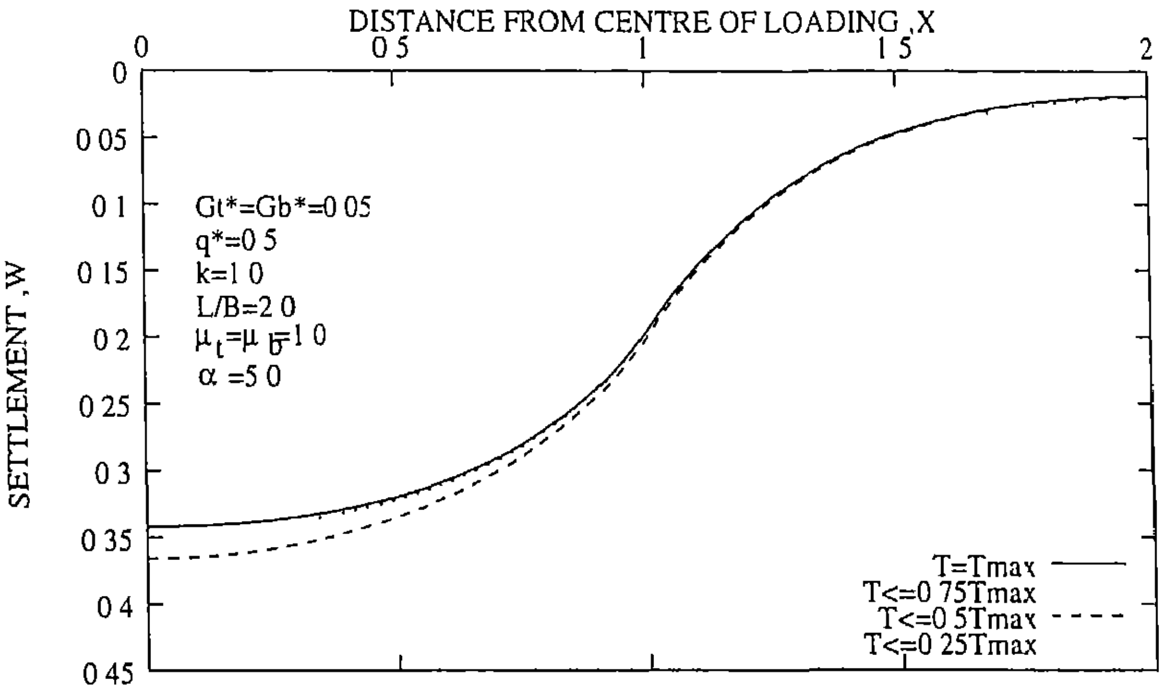

Figure 4 2 Settlement Profiles For $q^* = 0\ 5$ For Different Mobilised Tensions

Figs (4 2) and (4 3) show the settlement profiles at load intensity $q^* = 0\ 5$ and $q^* = 1\ 0$ respectively

From Fig (4 2) it can be seen that the effect of varying the mobilised tension has its effect upto the region L/B=1 25, and in Fig (4 3) upto the region L/B=1 5 as compared to Fig (4.1) in which this was only upto L/B=0 75 It can be seen

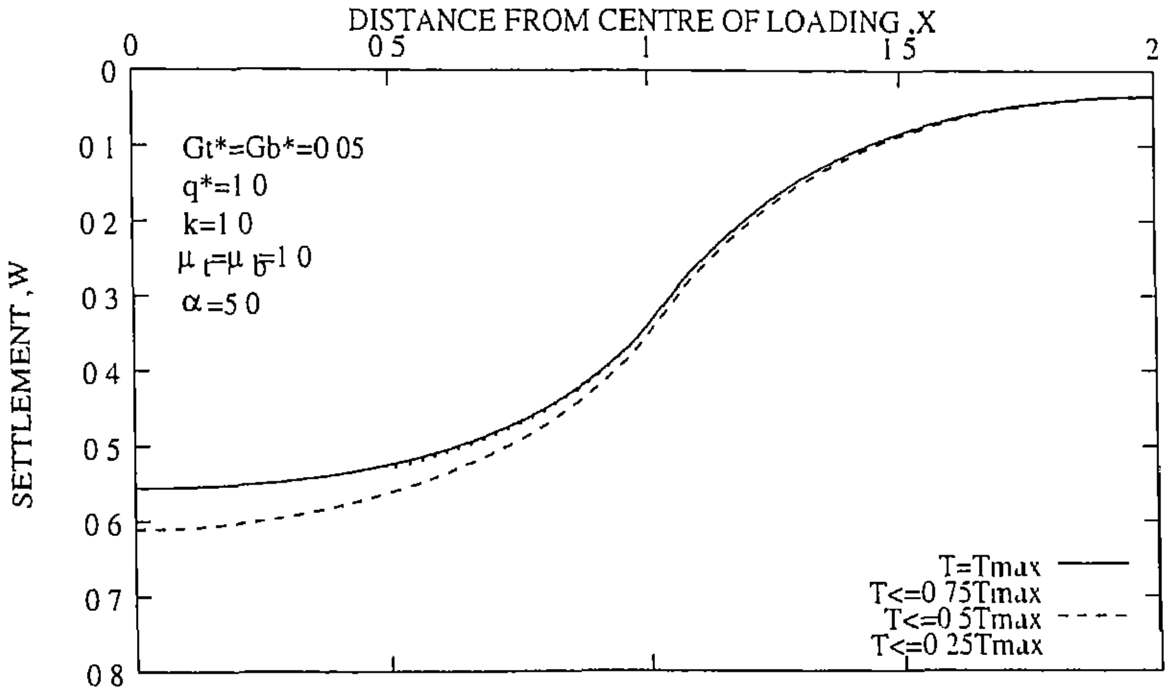

Figure 4 3 Settlement Profiles For $q^* = 1\ 0$ For Different Mobilised Tensions

from the Figs (4 1)-(4 3) that the effect of varying the mobilised tension on the settlement response of the system is pronounced at higher load intensities For example the increase in settlement for $q^* = 0\ 1$, $q^* = 0\ 5$ and $q^* = 1\ 0$ at the centre of loading for the case when the mobilised tension is limited to 50% of the maximum mobilised tension with respect to settlement in case of maximum mobilised tension is 2 2%, 7 1% and 9 9% respectively

4.3 Effect of Mobilised Tension For Different Values Of Lateral Stress Ratio

The effect of lateral stress on the settlement response of the reinforced granular fill soft soil system at different values of mobilised tension is presented in this section

Figs (4 1) and (4 4) show the settlement profiles at lateral stress ratios k=1 0 and k=0 6 respectively for $q^* = 0\ 1$ $G_t^* = G_b^* = 0\ 05$ $\mu_t = \mu_b = 1\ 0$, L/B=2 0 and $\alpha = 5\ 0$ From the figures it can be seen that settlement increases as the value of lateral stress ratio is reduced However at this low load intensity the effect of varying the mobilised tension is negligible

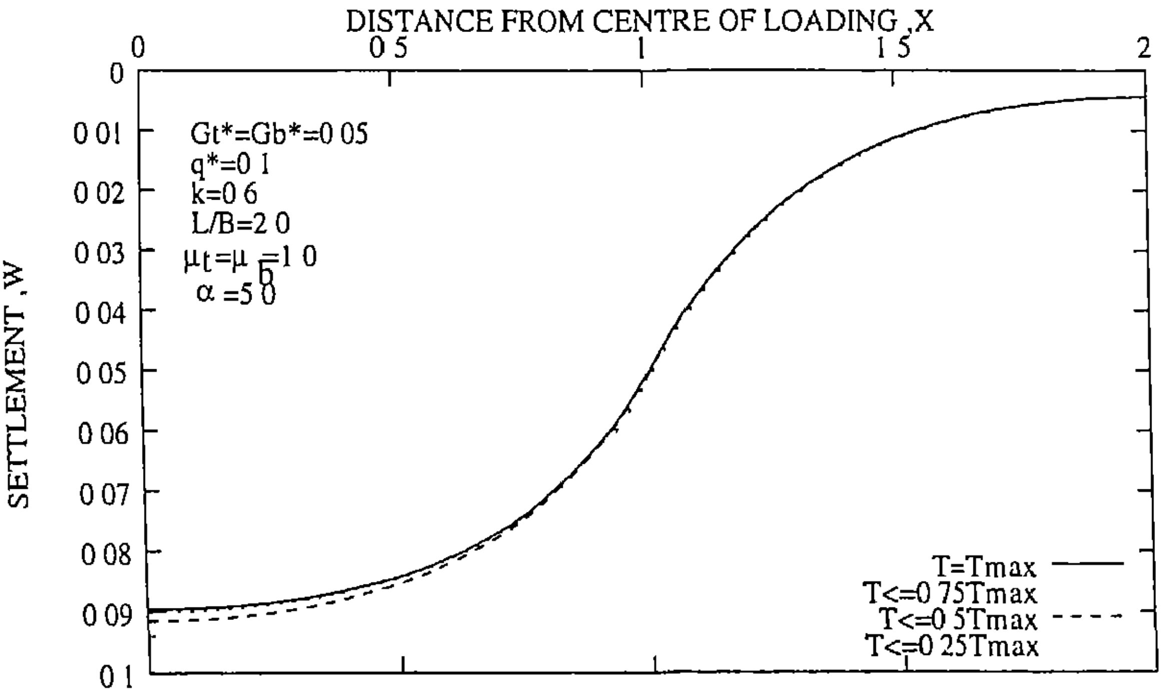

Figure 4 4 Settlement Profiles For k=0 6 And $q^* = 0\ 1$ For Different Mobilised Tensions

Figs (4 2) and (4 5) show the settlement profiles for lateral stress ratios k=1 0

and k=0 6 respectively at $q^* = 0\ 5$

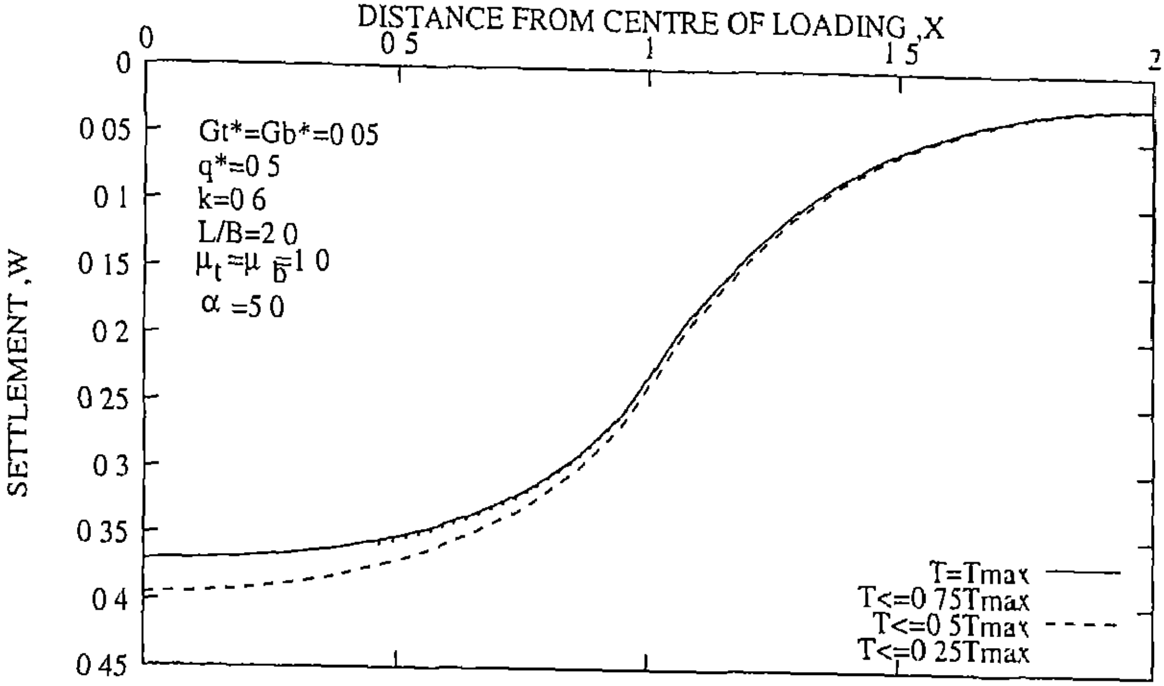

Figure 4 5 Settlement Profiles For k=0 6 And $q^* = 0\ 5$ For Different Mobilised Tensions

From these figures it can be seen that settlement increases with the decrease in the value of lateral stress ratio For example the increase in settlement at the centre of laoding for k=0 6 with respect to settlement at k=1 0 at $q^* = 0\ 5$ is 8 2%, 8 17% and 7 6% respectively for slippage at 75%, 50% and 25% of the maximum mobilised tension

Figs (4 3) and (4 6) show the settlement profiles for lateral stress ratios k=1 0 and k=0 6 respectively at $q^* = 1\ 0$

These curves also show similar trends as mentioned above However, it can be seen that at higher load intensity the effect of mobilised is more pronounced as

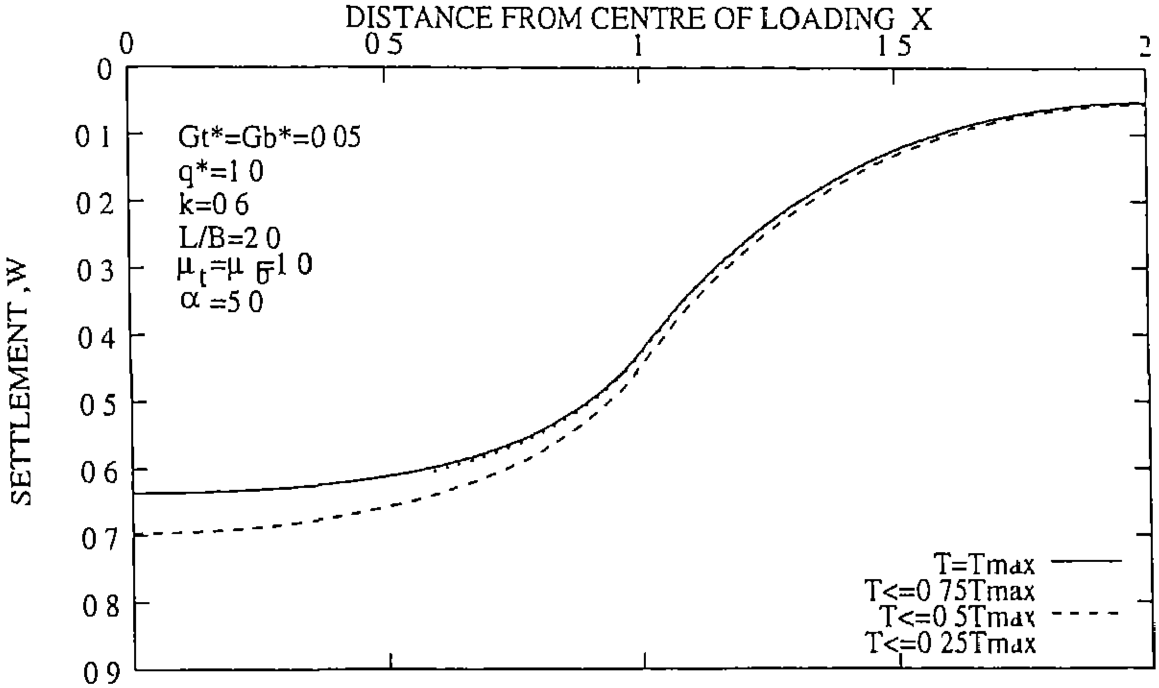

Figure 4 6 Settlement Profiles For k=0 6 And $q^* = 1\ 0$ For Different Mobilised Tensions

compared to that at lower load intensities

Thus from Figs (4 1)-(4 6), it can be observed that the effect of mobilised tension is more pronounced at higher lateral stress ratios For example at k=1 0 the increase in settlement at the centre of loading for $q^* = 0\ 1$ for the case where mobilised tension is limited to 25% of maximum mobilised tension with respect to maximum mobilised tension case is 5 1% Whereas for the same case at k=0 6 the inacrease in settlement is 4 9% The same trend is obtained at higher load intensities but the magnitude of increased settlements is higher As an example for the same case but at high load intensity $q^* = 1\ 0$ the increase in settlement at k=1 0 and k=0 6 is 32 08% and 31 9% respectively

$\alpha = 5$, and $\alpha = \infty$ respectively

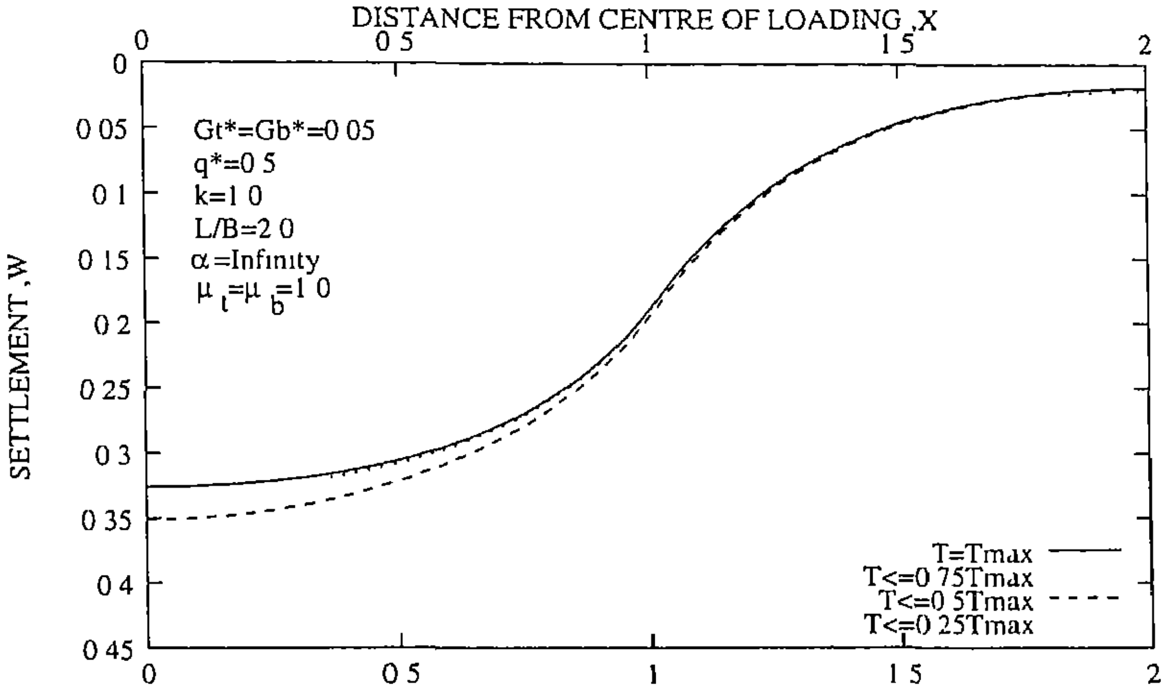

Figure 4.8 Settlement Profiles For $\alpha = \infty$ And $q^* = 0.5$ For Different Mobilised Tensions

From these two figures it can be noted that as the value of α increases from 5 to ∞ the settlement reduces by about 4.8% at the centre of loading for maximum mobilised tension case. Whereas for the cases where mobilised tensions are limited to 75%, 50% and 25% of the maximum mobilised tension the reduction in settlement is 4.7%, 4.2%, and 2.6% respectively. Figs. (4.3) and (4.9) show the settlement response of the system at $q^* = 1.0$ for $\alpha = 5$ and $\alpha = \infty$ respectively.

As the value of α is reduced from ∞ to 5 the settlement for the case of maximum mobilised tension and for the case where mobilised tension is limited to 75%, 50%, 25% of the maximum mobilised tension at the centre of loading is 6.4%, 6.3%, 5.9%

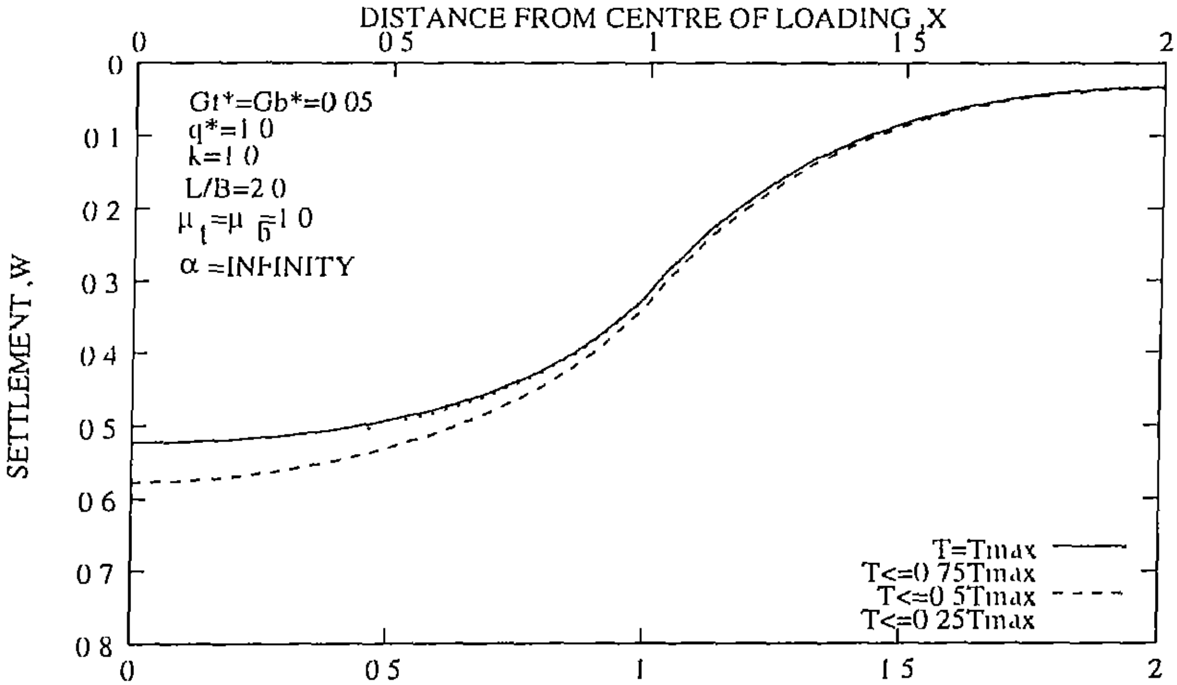

Figure 4 9 Settlement Profiles For $\alpha = \infty$ And $q^* = 1\ 0$ For Different Mobilised Tensions

and 4 08% respectively From Figs (4 1)-(4 3) and (4 7)-(4 9) it can be noted that at higher load intensity the magnitude of the effect of compressibility for different mobilised tensions and its subsequent effect on the settlement response of the system is high

4.5 Effect Of Mobilised Tension For Different Values Of Shear Modulus of Granular Fill

In this section the effect of shear modulus for different mobilised tensions and its subsequent effects on the settlement response of the system are presented Fig (4 10) show the setttlement profiles of the system at $G_t^* = G_b^* = 0\ 01$, $q^* = 0\ 1$,

$\mu_t = \mu_b = 1.0$, k=1 0 $\alpha = 5$ From the figure it can be seen that the effect of different mobilised tensions on the settlement response of the system is visible only in the region upto L/B=0 7 Beyond this region the effects of varying mobilised tensions are negligible

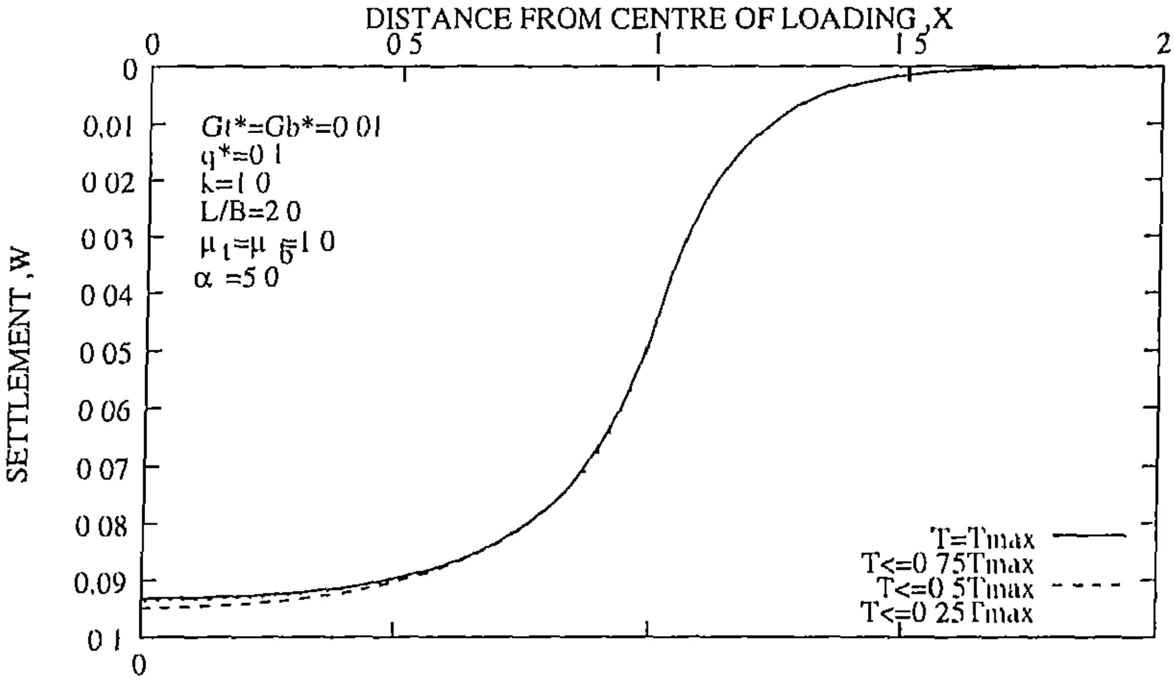

Figure 4 10 Settlement Profiles For $G_t^* = G_b^* = 0.01$ And $q^* = 0.1$ For Different Mobilised Tensions

Fig (4 11) show the settlement profiles for the system at $G_t^* = G_b^* = 1.0$ for $q^* = 0.1$ It can be noted that here also the effect of varying mobilised tensions is negligible on the settlement

Though on comparing the two figures it can be seen that the settlement is reduced by about 100% as the value of shear modulus is increased from 0.01 to 1 0 Fig (4 12) shows the settlement profiles of the system at $G_t^* = G_b^* = 0.01$ for $q^* = 0.5$ From

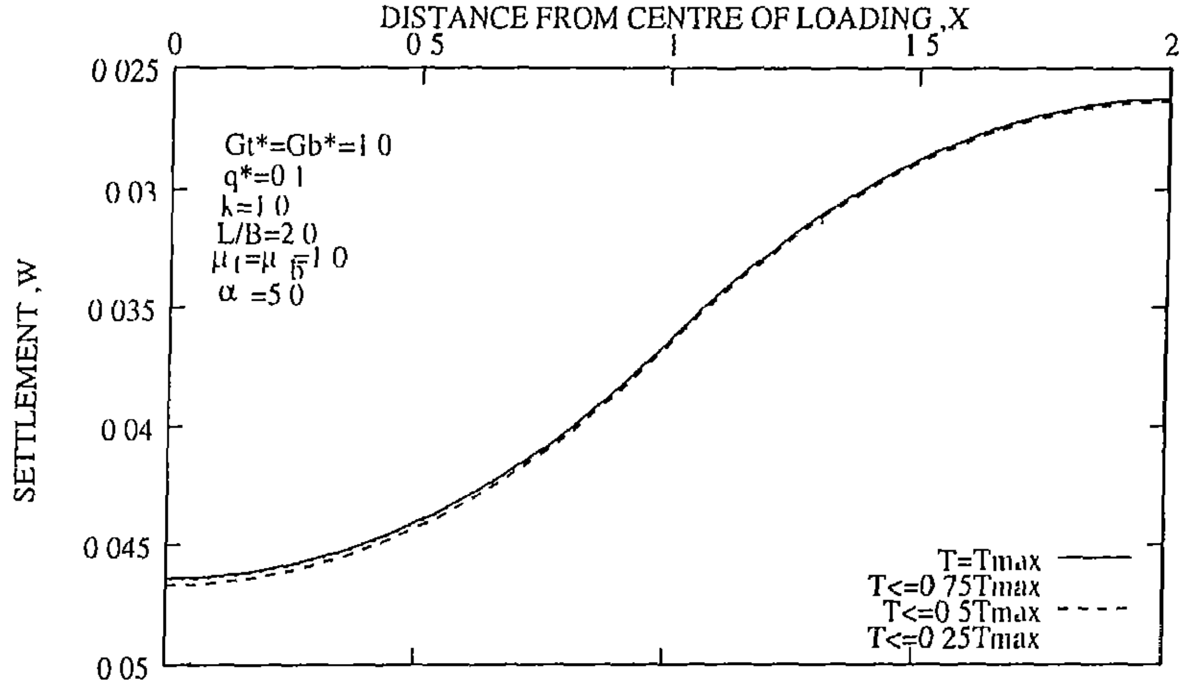

Figure 4 11. Settlement Profiles For $G_t^* = G_b^* = 1\ 0$ And $q^* = 0\ 1$ For Different Mobilised Tensions

the Fig it can be observed that the effect of shear modulus for different mobilised tensions on the settlement response of the system is only upto the region L/B=0 75

Fig (4 13) show the settlement profiles for $G_t^* = G_b^* = 1\ 0$ for $q^* = 0\ 5$ From the figure it can be seen that the effect of different mobilised tensions is to shift the settlement profiles Thus the effect of slippage is observed for the whole reinforced zone (i e upto L/B=2 0) From Figs (4 12) and (4 13) it can be observed that as the value of shear modulus is decreased from 1 0 to 0 01 the increase in settlement takes place For example the increase in settlement for maximum mobilised tension case and for the cases where mobilised tensions are limited to 75%, 50% and 25% of the maximum mobilised tension is 66%, 67 8%, 72 9% and 88 01% respectively

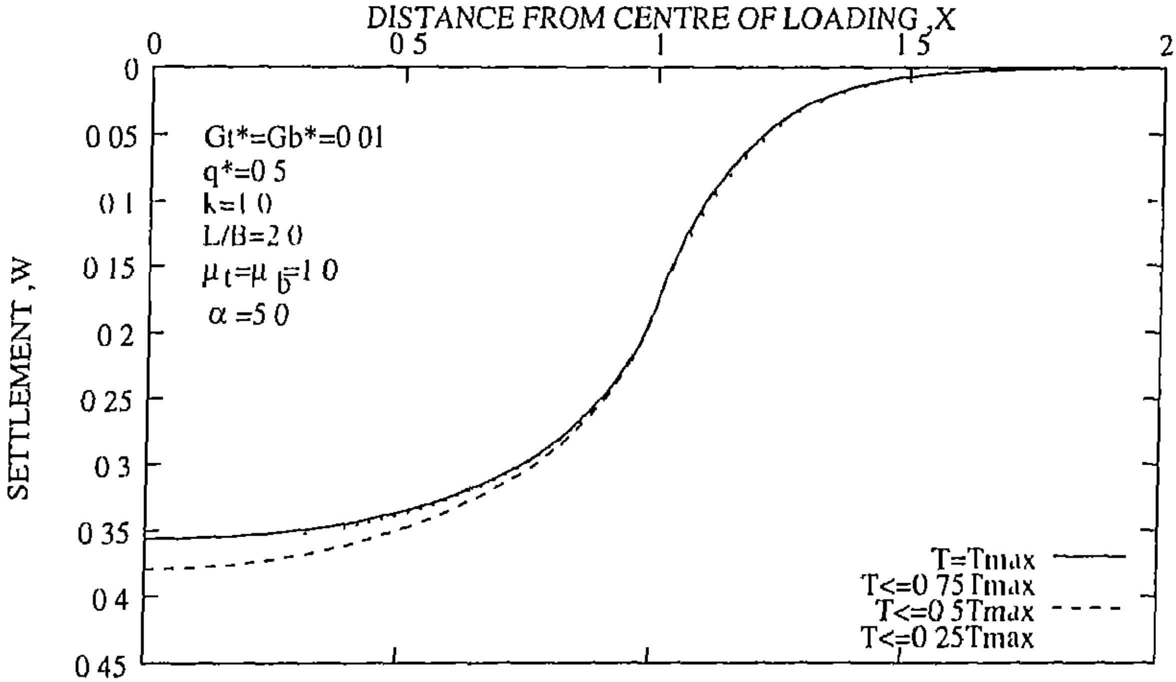

Figure 4 12 Settlement Profiles For $G_t^* = G_b^* = 0\ 01$ And $q^* = 0\ 5$ For Different Mobilised Tensions

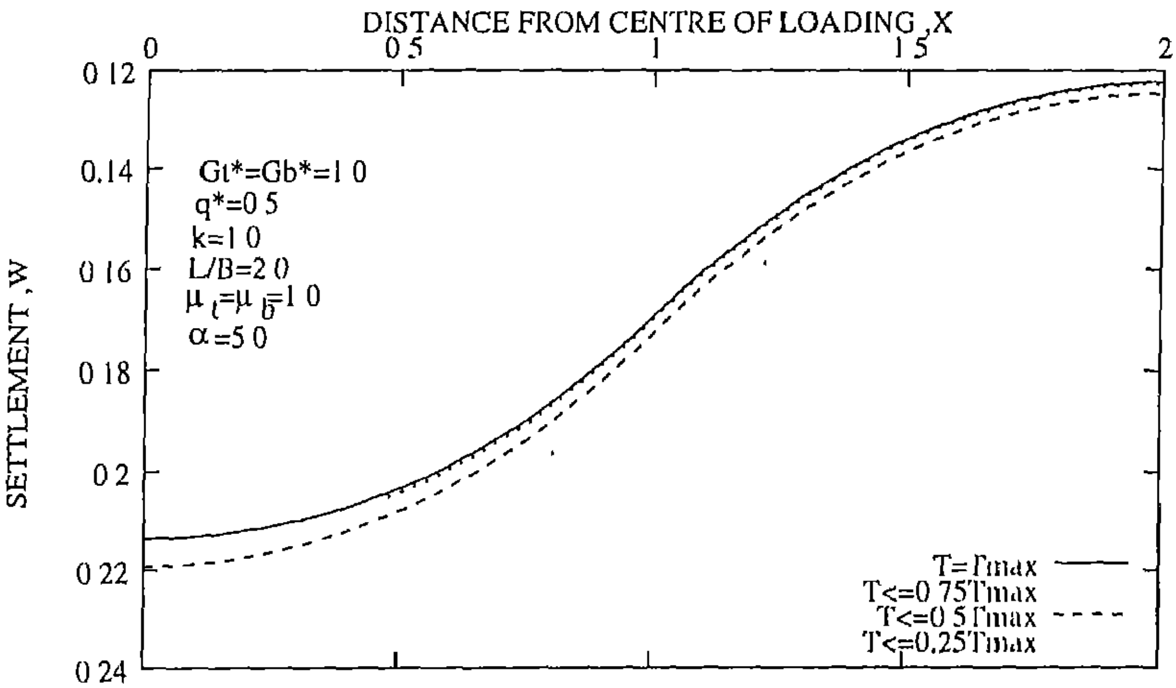

Figure 4 13 Settlement Profiles For $G_t^* = G_b^* = 1\ 0$ And $q^* = 0\ 5$ For Different Mobilised Tensions

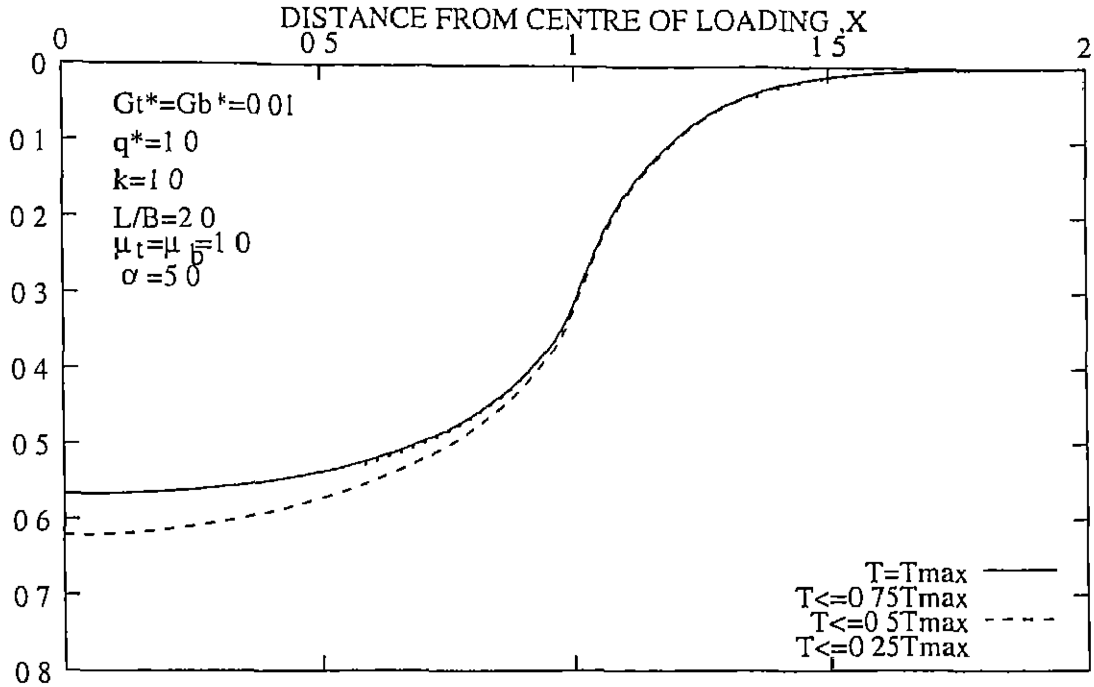

Figure 4 14 Settlement Profiles For $G_t^* = G_b^* = 0\ 01$ For Different Mobilised Tensions

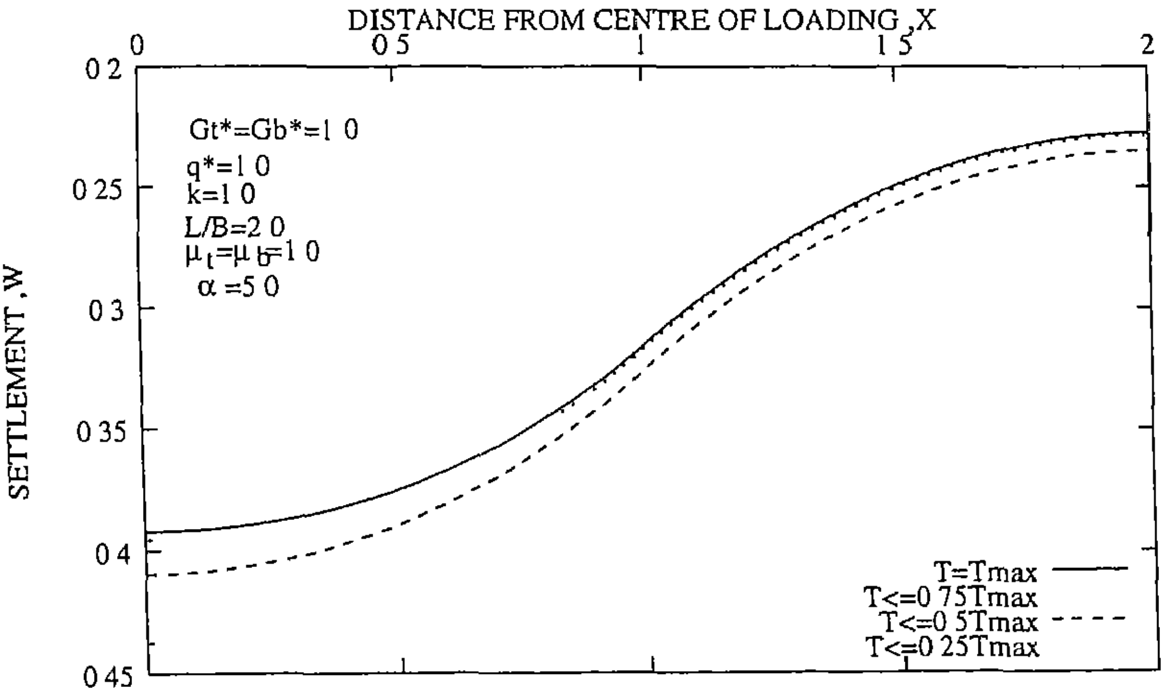

Figure 4 15 Settlement Profiles For $G_t^* = G_b^* = 1\ 0$ For Different Mobilised Tensions

Figs (4 14) and (4 15) show the settlement profiles of the system for $G_t^* = G_b^* =$ 0 01 and $G_t^* = G_b^* = 1\ 0$ respectively at $q^* = 1\ 0$ As the value of shear modulus is decreased from 1 0 to 0 01 the increase in settlement for maximum mobilised tension case and for the cases where mobilised tension is limited to 75% 50% and 25% of the maximum mobilised tension is 44 5% 45 7%, 51 6% and 72 5% respectively

4.6 Effect Of Mobilised Tension For Different Interfacial Friction Values

The effect of interfacial friction ratios for different mobilised tensions and its subsequent effect on the settlement response of the system is presented in this section Figs (4 16) and (4 5) show the settlement profiles of the system at $\mu_t = \mu_b = 0\ 1$ and 1 0 respectively for $q^* = 0\ 5$, $G_t^* = G_b^* = 0\ 05$, k=0 6 $\alpha = 5\ 0$ ang L/B=2 0

From these figures it can be observed that as the value of interfacial friction ratio is reduced from 1 0 to 0 1 the increase in settlement takes place For example for maximum mobilised tension case and for the cases where mobilised tensions are limited to 75%, 50% and 25% of the maximum mobilised tension the increase in settlement is 26 32%, 11 5%, 21% and 9 5% respectively

Figs (4 17) and (4 6) show the settlement profiles at $\mu_t = \mu_b = 0\ 1$ and 1 0 respectively for $q^* = 1\ 0$

From these figures it can be seen that the increase in settlement for for maximum mobilised tension case and for the cases where mobilised tensions are limited to 75%.

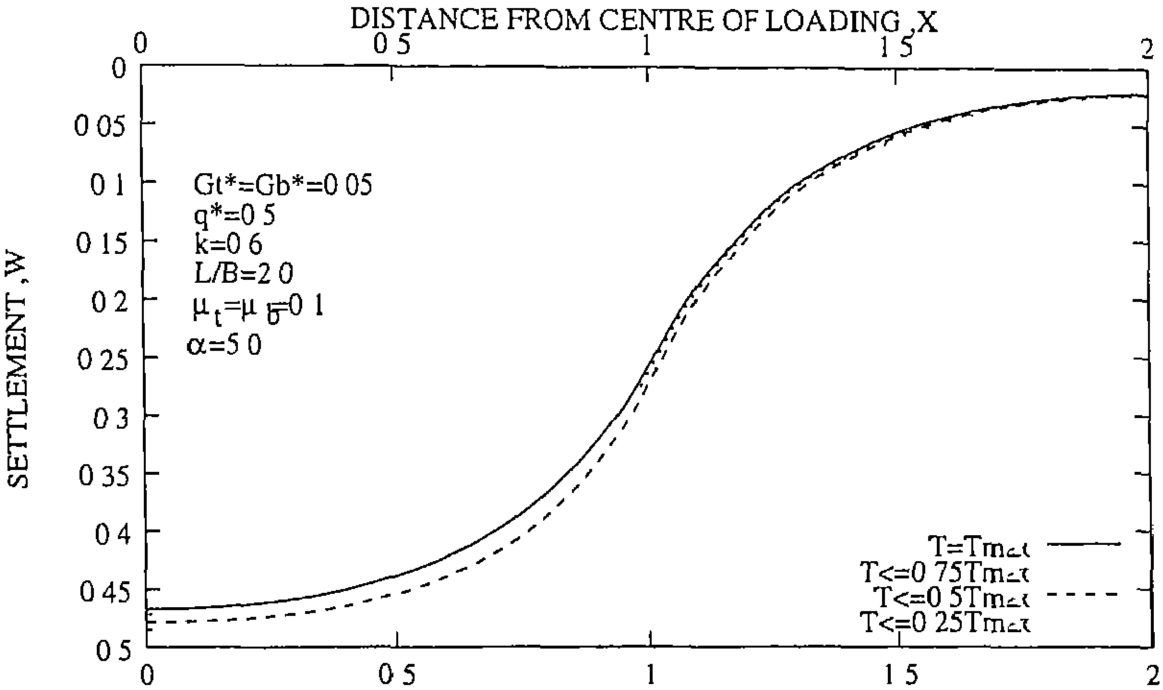

Figure 4 16 Settlement Profiles For $\mu_t = \mu_b = 0\ 1$ And $q^* = 0\ 5$ For Different Mobilised Tensions

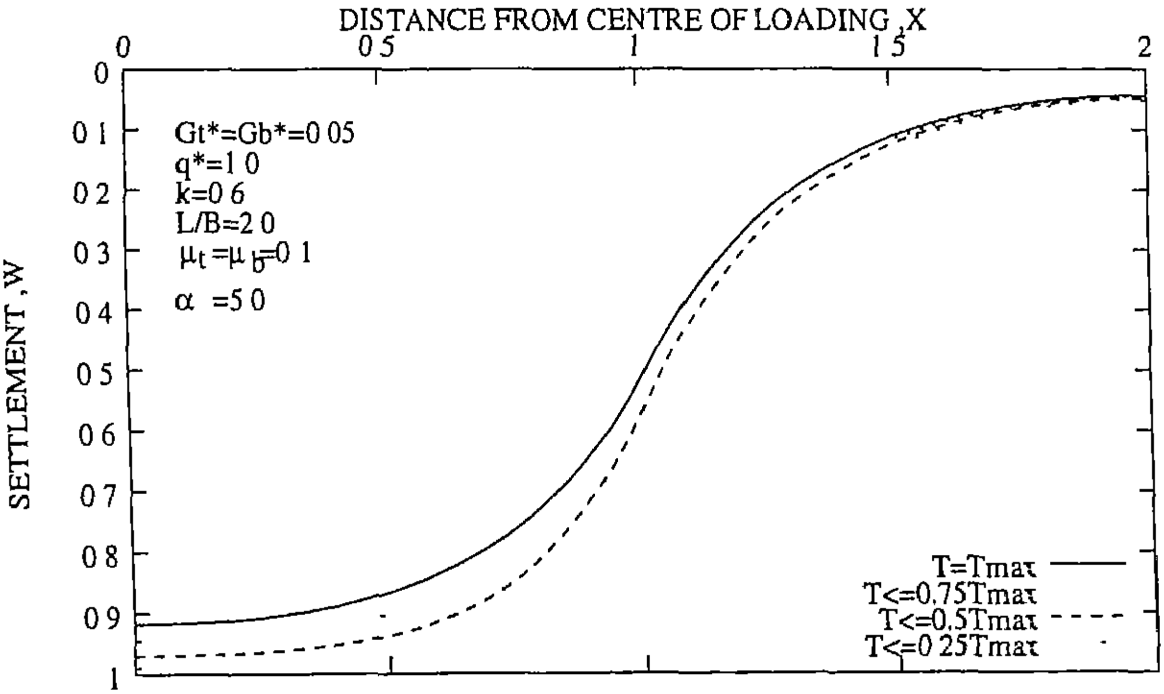

Figure 4 17 Settlement Profiles For $\mu_t = \mu_b = 0\ 1$ And $q^* = 1\ 0$ For Different Mobilised Tensions

50% and 25% of the maximum mobilised tension is 447 46 3%, 39/5% and 18 06% respectively

4.7 Conclusions

The suggested model has been successfully employed to determine the effect of different mobilised tensions on the settlement response of the system The variation of mobilised tensions have considerable effect on the settlement response of the system It is also observed that at higher load intensity the effect of varying mobilised tensions on the settllement response of the system is enhanced At smaller load intensity the effect of varying mobilised tensions on the settlement response of the system is observed only in the region close to the centre of the loading The various parameters also effect the settlement response of the system which has been discussed in the previous sections

Chapter 5

SUMMARY and CONCLUSIONS

With the availability of competent geosynthetics, their uses in many applications have become more common and have proven to be an effective means of soil reinforcement One such applications of geosynthetics is their use as reinforcement in foundation soils Geosynthetics are either placed at the interface of the granular fill soft soil or inside the granular fill

From the analysis of the results of large number of model tests conducted till very recently and the results presented through numerous analytical and numerical studies it has been observed that the effect of varying the mobilised tensions is neglected Therefore it was felt that this aspect needs consideration as it is going to effect the behaviour of the reinforced granular fill soft soil system With this in mind a model has been suggested in the present work which incorporates the variation of mobilised tension of the reinforced granular fill-soft soil system The equations which

govern the settlement response of the system have been developed by considering equilibrium of forces of shear layer and rough elastic membrane Finite difference scheme has been used to solve the governing equations The results are presented in nondimensionalised form Based on the results and discussions presented in earlier chapter the following generalised conclusions are drawn

- The suggested foundation model is well suited to study the effect of slippage of reinforcement on the settlement response of the reinforced granular fill-soft soil system

- The region effected by the slippage of reinforcement of the system is dependent on the load intensity At low load intensity the effect of slippage of reinforcement is only in the region upto L/B=0 75 from the centre of loading Whereas, at high load intensity this effect can be seen upto almost the entire length of the reinforcement

- At low load intensity the lateral stress ratio does not have much effect on the settlement response of the system

- At lower values of interfacial friction ratios there is an uniforn increase in the settlements due to the effect of slippage of reinforcement, whereas at higher values it is not the case

- The numerical apporach adopted is found to be very efficient in terms of economy of computations as it takes only few seconds of CPU time to obtain the settlement of the system

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